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Development of Pile Foundation Bias Factors Using Observed Behavior of Platforms During Hurricane Andrew

R.K. Aggarwal, and R.W. Litton, PMB Engineering Inc., C.A. Cornell, CAC Co., W.H. Tang, Hong Kong University of Science & Technology, J H. Chen, Chevron Petroleum Technology Co., and J.D. Murff, Exxon Production Research Co.

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Abstract

The performance of more than 3,000 offshore platforms in the Gulf of Mexico was observed during the passage of Hurricane Andrew in August 1992. This event provided an opportunity to test the procedures used for platform analysis and design. A global bias was inferred for overall platform capacity and loads in the Andrew Joint Industry Project (JIP) Phase I.[1] It was predicted that the pile foundations of several platforms should have failed, but did not. These results indicated that the biases specific to foundation failure modes may be higher than those of jacket failure modes. The biases in predictions of foundation failure modes were therefore investigated further in this study. The work included capacity analysis and calibration of predictions with the observed behavior for 3 jacket platforms and 3 caissons using Bayesian updating. Bias factors for two foundation failure modes, lateral shear and overturning, were determined for each structure. Foundation capacity estimates using conventional methods were found to be conservatively biased overall.

Introduction

Hurricane Andrew was a very intense storm that passed through the Gulf of Mexico in August 1992. While most of the Gulf of Mexico platforms were not adversely affected by Andrew, twenty eight steel jacket platforms were significantly damaged. All of these were installed during 1948 to 1969 and were located in water depths up to 143 feet. In addition, forty seven caissons, installed during 1979 to 1991 in water depths up to 113 feet, were also significantly damaged. While foundation failure was identified as the primary cause of damage to most caissons, it was identified as the cause for damage of only one steel jacket platform.[1,2]

An extreme event such as Andrew provides a unique opportunity to study offshore structures tested under full scale field conditions. By reviewing the platforms that survived, were damaged, or failed during the hurricane it is possible to improve our understanding of the behavior of platforms during large storms. In October, 1993, Phase I of a joint industry project, "Hurricane Andrew — Effects on Offshore Platforms,"[1,3] was completed. This project established that there was bias in the safety factor (capacity to load ratio) by combining the analytical and observed behavior of 13 jacket platforms. The inferred bias is characterized as a correction needed to bring analytical results to agreement with the observed results.

The capacity analysis performed in Phase I of the Andrew JIP indicated that failure of foundation elements should have occurred in a majority of platforms analyzed. Since this was not observed during post-Andrew inspections, it was concluded that the deterministic analysis was conservatively biased. An overall (system) correction factor, with a mean of 1.2 was established. This correction factor was not specific to any particular failure mode in the jacket structure or its foundation but rather included failures of all types.

Historically, very few jacket platforms are known to have failed due to foundation weaknesses. If not properly accounted for this conservatism in estimates of foundation capacity can lead to misinterpretation of important failure modes in the jacket. Therefore, it is doubly important to use the experience from behavior of jacket foundations during Andrew to improve our understanding of biases in the foundation capacity estimates.

To investigate the bias in the foundation capacity estimates for steel jacket platforms, the American Petroleum Institute (API) and the U.S. Minerals Management Service (MMS) commissioned a project specifically to evaluate the behavior of offshore platform foundations during Andrew. This paper presents the approach followed and the results obtained in the API/MMS Foundation Study completed in May 1995.[4] This project initially had two primary objectives.

1. To perform a calibration of procedures used for foundation capacity analysis (lateral and axial capacities) in assessment of existing platforms. The task included reconciling analytical predictions of platform damage and failures with observed field performance during Andrew and thereby determining biases in the analytical predictions.

2. To identify the various parameters which influence the bias factors

The determination of more realistic estimates of platform capacity has become increasingly important, due to new guidelines for assessment of existing platforms included in the Draft Section 17, an addendum to the API RP 2A, twentieth edition[5] This section allows older platforms to meet reduced criteria in the ultimate limit state from those used in new design. This document allows the use of nonlinear static pushover analysis as the most sophisticated procedure to establish the ultimate capacity of platforms and to evaluate their likelihood of surviving large storms in their intact or damaged state. This procedure is significantly different than the conventional method followed at the design stage.

Approach

The calibration approach involves a comparison of analytically predicted platform performance to observed platform performance. The end result is a probability density function describing the prediction bias that can be used to improve understanding of platform safety

In principle, it is possible (with enough data) to "calibrate" the various parameters in the capacity and load prediction algorithms for steel jacket platforms, that is to determine correction factors to their nominal estimates. Examples of parameters are wave height and drag coefficient on the loading side or material yield and soil strength on the structural capacity side. The available information from post Andrew field inspections of platforms was too limited to make this approach feasible. It was concluded that the approach consistent with available data was to evaluate the correction to overall platform safety factors (platform capacity to load ratio). The correction was defined as a bias factor "B" applicable to the computed "safety factor" of the platform:

$$(R/S)_{true} = B(R/S)_{computed} \qquad (1)$$

where R represents resistance or strength and S represents loading. B is a random variable. A mean value of B greater than 1.0 indicates that (on the average) the current ultimate capacity analysis procedures provide conservative results. A mean value of B less than 1.0 indicates (on the average) unconservative results. Failure is associated with $B\,(R/S)_{computed}$ being less than 1.

In this project two bias factors, applicable to the foundation lateral (B_{fl}) and foundation axial (B_{fa}) capacity to load ratios of platforms, were determined using the calibration methodology shown in Fig. 1. The methodology consists of the following three stages of analysis

- Capacity analysis
- Reliability analysis
- Bayesian updating

Capacity analysis yields the mean lateral load levels at which successive inelastic or failure events would occur in the jacket and its foundation elements. The mean ultimate capacity is defined by the maximum lateral loading which can be resisted by the platform.

The reliability analysis determines the probability of occurrence of the observed event (failure, survival, or damage) by combining the observations with capacity analysis results, hindcast data, and uncertainties in various parameters. The failure probabilities can be estimated using a rigorous probabilistic approach (see Appendix).

The Bayesian updating procedure combines a "prior" distribution of the bias factor (based on estimates of the safety factor bias before Andrew) and the new information from behavior of platforms during the hurricane (which is included in the form of probabilities of occurrence of observed behavior) By combining the two using standard Bayesian methods,[6] the updated or "posterior" distribution of the bias factor is obtained.

This calibration methodology was developed in Phase I of the Andrew JIP[1,3] It is discussed herein as it was applied to determine the characteristics of the two bias factors applicable to the foundation failure modes.

Platforms Investigated

Three steel jacket platforms out of 13 platforms investigated in the Andrew Phase I were selected for evaluation in this study. Their selection was based upon review of their configurations, availability of geotechnical information, and expected utility to the calibration based on predicted vs. observed behavior from Phase I analysis. None of these platforms was known to have observable damage to their foundations from Andrew. However, it has been determined that a foundation could have technically experienced failure without a visible manifestation. Therefore, additional calibration cases were introduced in this study, to evaluate the sensitivity of varying interpretations of the observations (failure vs. survival for example)

In lieu of any jacket platform cases with known foundation damage available to this study, three caissons with suspected foundation damage during Andrew were selected for investigation in this study. They were included to assess the impact on the foundation lateral bias factor (B_{fl}) of damage cases. Because of the observed failures, the caissons provided an opportunity to improve the bias estimate for the lateral foundation capacity predictions.

Foundation Capacity

The goal of this study was to determine bias factors specific to the capacity estimates for lateral and axial foundation mechanisms. Therefore, it became necessary to analyze additional cases for each platform to isolate the effects of uncertainties in the various mechanisms. The objective was to obtain relatively uncoupled estimates of axial (overturning) and lateral foundation capacities for use in calibration Three analyses were performed for each platform using the following models

Base Case analysis	Nonlinear jacket and foundation model — To determine the critical failure mode for the best estimate deterministic model
Case 2 analysis	Linear jacket and nonlinear foundation model (jacket and axial foundation failures suppressed) — To estimate the ultimate capacity of the platform associated with the pile yielding/hinging (lateral) mechanism.
Case 3 analysis	Linear jacket and nonlinear foundation model (jacket and lateral foundation failures suppressed) — To estimate the ultimate capacity of the platform associated with overturning i.e., the pile pull-out/plunging (axial) failure mechanism.

The Case 2 and Case 3 analyses provided separate estimates of load levels corresponding to failure events for the two pile-soil mechanisms in the pile/soil system, by elimination of failures predicted in the jacket elements, and by elimination of the axial modes in Case 2 and the lateral modes in Case 3.

In the Case 2 analyses, the lateral loads which initiated first yield and full plasticity of a pile section and subsequently full plasticity in several piles leading to development of a failure mechanism were determined. In the Case 3 analyses the lateral loads which initiated pullout or plunging of the first pile and subsequently of several piles leading to a failure mechanism formation were determined. The ultimate capacities were defined as the load levels at the complete formation of failure mechanisms.

In the case of caisson platforms, the load levels at first yield of a section, at the fully plastic section, and at ultimate capacity were established. The ultimate capacity for some caissons with non-compact sections is limited by local buckling of the tubular sections upon development of a fully plastic section and hence the two conditions are synonymous.

Static Pushover Analysis. The ultimate capacity estimates were determined using static pushover analysis, which is a common approach used by the industry to determine the maximum lateral load carrying capacity of an offshore platform.

The static pushover consists of a representative "snapshot" of lateral wave forces acting on the platform, including any wave forces acting on the deck. These forces are then scaled up monotonically until platform collapse is predicted. The corresponding base shear acting on the platform at time of failure is defined as the platform capacity. This load can then be related to a specific wave height that causes platform failure. Further descriptions of the static pushover method can be found in several references [3,7,8]

Such analysis requires modeling of nonlinear material and geometric behavior of various elements of a platform. Special nonlinear elements are used to mimic the behavior of the jacket braces, legs, piles and soils In this analysis all aspects of loading and response are modeled based on expected behavior, thus any known biases in their formulations are removed. The mean estimates of component strength are used instead of the lower bound estimates recommended in the 20th Edition of API RP 2A for new designs. In this project, the loading was determined using hindcast metocean data and the procedures given in the API RP 2A. A simplified procedure given in the Draft Section 17 supplement to the API RP 2A was used to determine the forces due to waves inundating the deck. The computer program CAP[9] (Capacity Analysis Program) was used to perform this analysis.

The nonlinear computer model used for the static pushover analysis of an 8-leg steel jacket platform is shown in Fig. 2. The model consisted of a fully coupled nonlinear jacket-foundation system. The force-deformation relationship used to model each of the primary elements is shown The piles were modeled as beam-column elements which carry both bending and axial loads Nonlinear p-y (lateral), t-z (axial skin friction) and q-z (axial end bearing) springs were attached to pile nodes to model pile/soil behavior.

The fully coupled nonlinear pile-soil system computer models were also developed for caisson platforms. The caisson and its foundation were divided into a number of nonlinear beam-column elements. The deck structure and boat landing framing were modeled as linear-elastic beam elements. The soil was represented by nonlinear p-y and t-z springs.

Some of the items which are important to determine the estimates of platform capacity are discussed below. The various factors which influence soil shear strength and estimates of lateral and axial soil capacities are identified.

Material Strength. All the platforms analyzed in this study were fabricated using steel with a 36 ksi nominal yield strength. A yield strength of 42 ksi was used to account for the difference between nominal and mean yield strength and to account for increase in strength due to strain rate effects (rapid loading in storms) [10]

Soil Shear Strength. The soils at the locations of the structures studied are clays which increase in strength with depth. In relatively shallow water these clays are typically overconsolidated near the seabed. In some cases intermittent sand layers exist, however, these were generally sufficiently thin and deep as to have little influence on lateral or axial pile behavior

The soil strength data available ranges in vintage from the sixties to the late eighties, further, some of the borings were located at considerable distances from the platform sites. For these reasons the soil data quality varied significantly and considerable judgment was required in the selection of a strength profile for use in analysis.

In keeping with findings from previous studies,[11,12] the idealized strength profiles were based primarily on miniature vane tests on "undisturbed samples" where possible. However, the following were also considered in its assessment.

- The strength ratio (S_u/σ_v') for typical Gulf of Mexico normally consolidated clay (= 0 23)
- The method of sampling - pushed or driven
- Other strength tests available along with typical "modification factor" - UU and UC
- Strength data represented in the API pile load test data base

The report to API/MMS[4] contains more details on the strength profile selection including specific profiles used.

Soil Elements. The soil resistance is represented by soil springs which are characterized by nonlinear p-y (lateral), t-z (axial shaft), and q-z (axial end bearing) curves given in API RP 2A, 20th edition These were modeled using the PSAS (pile-soil analysis system) suite of elements included in the CAP program. The static API RP 2A capacities were modeled for both p-y and t-z springs.

Lateral Soil Capacity. In this study the lateral p-y nonlinear springs, attached to the pile nodes, were modeled using the static capacity estimates given in the API RP 2A as opposed to the cyclic p-y springs used in new design.

Recent model tests[13] have indicated that for pushover type analysis, the static lateral soil criterion provides a better ultimate capacity prediction. This is because the displacements of piles at ultimate loading are significantly greater than the typical test displacements on which the API RP 2A p-y behavior is based. Thus, it is likely that the extreme loading would cause the pile to close any existing gaps and deform virgin soil that has not been previously degraded. This behavior is believed to result in very conservative results where the cyclic curves are used.

The above findings are relatively recent and have not yet become established practice nor generalized to a wide range of conditions. Nonetheless for assessment of existing platforms against overloading from hurricanes, the contribution of such effects should be considered in determining unbiased estimates of ultimate capacity of the foundation system.

Axial Capacity of Piles. Nonlinear t-z springs which are attached to the pile nodes were based on static axial capacity per API RP 2A, with no reduction in the capacity.

A review of the previous research and pile load tests indicated that large uncertainties exists in estimating the contributions of individual factors which influence axial capacity estimates. For example, the contributions of loading rate (or strain rate effect), cyclic loading, reconsolidation (time effect), and aging effects, which influence pile axial capacity are not reliably quantified and can have compensating effects on pile axial capacity. Therefore, they were not explicitly included in t-z modeling The recommended factors of safety contained in RP 2A implicitly recognize these effects, the importance of which varies from platform to platform. If any of these effects were to be explicitly taken into account, then the other effects would also need to be explicitly included to be consistent.[14]

Because of the large variabilities involved, in this study no attempt was made to interpret the effect of individual factors other than pile flexibility on the pile axial capacity estimates Pile flexibility (pile length effect) effects were explicitly accounted for in the capacity analysis using CAP, which models the loss of skin friction at large pile displacements The effect of other factors will be reflected in the bias factor.

Prior Knowledge of Biases in Foundation Capacity

The results of field tests of laterally loaded piles and analytical predictions were compared in an API PRAC project,[15] and the model bias and errors associated with the API p-y static and cyclic curves were determined. This work indicated that the current procedures predict maximum moment response more accurately than pile head displacements for working stress levels. The soft clay criterion was found to provide the best p-y curve for predicting lateral response of piles in clay Mean correction factors (biases) of 0 92 (fixed pilehead case) and 1 08 (free pilehead case) were determined for the lateral pile response (maximum bending moment) associated with the static loading p-y curves for the clay The corresponding COVs (errors in bias) were recommended as 0.20 (fixed pile head case) and 0 09 (free pile head case)[15,16]

In another API PRAC project, the biases associated with the API RP 2A pile axial capacity models were determined using a probabilistic model.[17] The overall bias associated with RP 2A, 16th edition procedure was estimated to range from 1.3 to 3 7 and the corresponding COVs from 0.32 to 0.53 depending on how the undrained shear strength was determined at a given site. A major contributor to the bias was found to be the loading rate effect which alone has an estimated bias of 1.56 It was suggested that the model errors will reduce with new capacity prediction models included in the 17th edition of RP 2A, and the overall bias is more likely to range from 1.5 to 3 0 with associated overall COVs from 0.3 to 0.4[18]

Based on the API PRAC studies, the prior distribution of the bias factor, associated with the lateral capacity estimates of a jacket foundation using the API static p-y curves, was assumed to have a normal distribution with a mean of 1 0 and a COV of 0 3. The prior distribution of the overall bias in the factor of safety for pile axial capacity was assumed to have a normal distribution with a mean of 1.3 and a COV of 0 3, applicable to piles primarily in clay.

Considering the relatively poor quality of the data it was the consensus of the project advisory group that values of 1.5 and above were overly optimistic. These values were believed to provide a reasonable balance between the effects of loading rate, cyclic degradation and choice of undrained shear strength of soil.

Calibration Process

The objective of calibration is to update the prior distribution of "B" (B_ℓ or B_a) in a manner consistent with the observed behavior of platforms during Andrew. The calibration is achieved by using Bayes theorem from probability theory[6] which states.

$$f''_B(b) \propto f'_B(b)\, lk(b \mid new\ information) \quad \text{(2)}$$

in which $f'_B(b)$ is the "prior" distribution of bias factor, B, $f''_B(b)$ is the "posterior" distribution, and lk (b| new information) is the "likelihood function" which reflects the information about B contained in the new observation. The likelihood function depends upon the observed state of a platform, i.e., survived, damaged, or failed during Andrew. This process is carried out for both lateral and axial failure modes independently.

Likelihood Function. The likelihood of the bias factor B less than b, given an observed failure of a platform due to a storm approaching from a particular direction is represented as follows

$$\begin{aligned} lk(b \mid failure) &= P[failure \mid b] \\ &= P_f(b) \quad \text{(3)} \end{aligned}$$

in which $P_f(b)$ is the probability of failure of a platform at B = b. This case was used for caisson platforms since failure was observed in each case.

For a platform survival case (no observed damage), the likelihood function becomes:

$$\begin{aligned} lk(b \mid survival) &= P[survival \mid b] \\ &= 1 - P_f(b) \quad \text{(4)} \end{aligned}$$

This was used for jacket cases, for which no damage to foundation elements were observed. For a damaged platform case (e.g., pull out or plunging of one or more piles; development of a hinge in pile sections), the likelihood function is the probability that the observed damage lies in the interval of the capacity to load ratio as predicted by the pushover analysis. The predicted ratios corresponding to the observed damage and the subsequent damage state (e.g., axial failure of one more pile or development of a single hinge in pile sections) are denoted by α_1 and α_2 respectively. The resulting likelihood function for a damage platform case would be:

$$lk(b \mid damage) = P_f(\alpha_1 b) - P_f(\alpha_2 b) \quad \text{(5)}$$

The above likelihood functions, for a range of values of "b," represent the information about bias factors (B_ℓ and B_a) contained in the observed behavior of an individual platform. Additional description of likelihood functions is given in OTC 7473[3] and OTC 8077.[19]

The combined likelihood functions of B_ℓ or B_a given the observed behavior of a number of platforms with a combination of survivals, damages, and failures is obtained by direct multiplication of the likelihood curves for each of the individual platforms as follows.

$$lk(b \mid n\text{-}observations) = \prod_{platform}^{n} \left[lk\,(b \mid observation\ i)\right] \quad \text{(6)}$$

Probability of Failure. The probability of failure (Eq 4) was estimated for each platform for different "b" values assuming $P_f(b) = P[\,bR/S < 1]$, in which R and S are considered as random variables. The failure probability formulation used is given in the Appendix. The probabilities were determined using hindcast data for storm hour segments with high wave heights. The 1992 Andrew hindcast[21] was used in this study.

The load, S, represents the maximum load on the structure during Andrew. The load is represented by the base shear, *BS*, obtained for different combinations of wave height (h) and current (u) by an empirical formulation given in the Appendix. The best estimate of the capacity (R) is represented by the ultimate capacity of a platform obtained from the static pushover analysis.

The uncertainties and distributions of various quantities in the equations in Appendix A are required for evaluating the probability of failure. The distributions and variances given in Table 1 were used in this project. Correlations between seastates, load levels, and capacities were neglected.

Bias Factors. The combined likelihood functions developed for a number of platforms in Eq. 6 are then used to establish distribution of two bias factors, B_ℓ and B_a, using Eq. 2. The mean values and COVs of the posterior distributions are then determined. The posterior mean values of Bs identify biases (conservatism or nonconservatism) in the analysis procedures. The shift from the prior to the posterior mean represents the effect of information coming from the Andrew experience. The COVs represent the remaining uncertainty in the true value of the two bias factors.

Application to Platforms

The calibration process was applied to 3 steel jacket platforms and 3 caisson platforms. Of 3 jacket platforms, none was observed in inspection to have foundation damage due to Andrew, whereas the structures of two of these were significantly damaged. Of these three, two had predicted damage to the foundation. One jacket platform was predicted to have no damage in either jacket or foundation and in fact had none. All 3 caissons were damaged due to Andrew.

A summary of the key features of these platforms is given in Table 2. The undrained shear strength of the soil varied from 0.5 to 0 7 kips/sq ft. at the mudline and from 1.1 to 1.6

kips/sq ft. at 100 ft. below for jacket platform locations. The caissons were located in weaker soil, than for the jacket platforms, with undrained shear strengths of 0.2 to 0.5 ksf at the mudline and from 0.6 to 1.2 ksf at 100 ft. below the mudline.

Capacity Analysis. The site specific hindcast data for all platforms was developed; it indicated a sharp reduction in significant wave heights beyond the most intense 2 to 3 one-hour segments of the storm (which were within ±10 degrees from the direction of the maximum storm). Therefore, the capacity analysis for only one direction was considered adequate.

Both 8-leg platforms have similar jacket configurations and pile details (see **Fig. 2**), and their ultimate capacity estimates for the Base Case analysis are 4,000 kips and 3,200 kips. The Case 2 lateral foundation capacity estimates for these two platforms are almost identical: 5,000 kips for platform P-1 and 5,100 kips for platform P-2. The estimates for Case 3 analysis are the same as for the Base Case. The lateral capacities are estimated to be 25 to 60 percent higher than the axial capacity estimates for the foundations.

The soils at platform P-2 are weaker than those at platform P-1, but the lateral ultimate capacity estimates are essentially equal. This results because the lateral capacity depends on the plastic moment capacity of the piles as well as soil shear strength.

Platform P-3 is a 4-leg platform in 62 ft. water depth, which experienced significant damage to the jacket as a result of Hurricane Andrew. The damage consisted of failure of K and KT joints. The hindcast estimated a maximum wave height of 51.5 ft for this location, which resulted in a total expected peak loading of 1,800 kips. The Base Case analysis predicted ultimate capacity as 1,600 kips. The Case 2 and Case 3 ultimate capacity analysis results were 2,400 kips and 2,500 kips, which on the average are higher than the maximum loading. The first plastic hinge was predicted at 2,100 kips, which are higher than the expected value of the predicted maximum loading.

The predicted system factor (ratio of load level at failure of several piles/ lateral load at failure of first pile) for the lateral capacity (plastification of pile sections) of pile/soil system was estimated as 1.14 to 1.18, which indicates load redistribution among piles for both 4- and 8-leg jacket cases. The predicted system factor for pile axial capacity was estimated between 1.2 to 1.3 for both 8-leg jackets, indicating that the load can be redistributed from the overloaded to the underloaded piles. In the case of the 4-leg jacket, the analysis indicated that no load redistribution was possible (axial capacity system factor = 1.0).

Calibration. The Case 2 and Case 3 capacity analyses results were used in calibration work to determine two foundation bias factors (lateral, B_n and axial, B_a). The capacity analyses indicated that foundation failures (lateral and axial) can occur at relatively small displacements (perhaps unobservable by divers). Therefore, several possible alternative states were investigated to consider the uncertainties in interpreting observations from field inspections.

Foundation Lateral Capacity. The foundation lateral bias factor (B_n) was determined for the following three possible interpretations of no observable damage to the foundations:

Case A. Fully plastic section event did not occur in any pile
Case B Fully plastic section events did not occur in several piles
Case C. Fully plastic section event did occur in one pile and others were undamaged

The likelihood functions and posterior distributions were developed for these three cases. **Fig. 3** shows the likelihood functions for jacket platforms and caissons for Case A. The prior and posterior distributions of B_n for Case A are shown in **Fig. 4** The shift in B_n is similar for both 8-leg platforms The platform P-3 has a minimal effect on the bias factor.

The resulting posterior distributions are summarized in **Table 3.** The mean values of the posterior distributions of B_n were similar for both Case A and Case B (Case A, mean B_n = 1.32 and Case B, mean B_n = 1.26) The mean value of the posterior distribution of B_n for Case C was the same as that assumed for the prior distribution (i.e., 1.0). The mean B_n for Case A and Case B reduced by 17% when the effect of three caissons (which had unequivocal foundation damage during Andrew) was included. Note that one caisson analysis indicated no damage would occur contrary to the observed performance and thus has unconservative bias. The effect of caissons for Case A are shown in **Fig. 5.**

It is clear that the inferred biases in the lateral failure mode are significantly different for platforms and caissons It is not clear why this occurs. There are important differences in the two structure types which may impact this result. This is an area that needs further study but in the interim one should consider the following in assessing these results

- The platform piles have significant redundancy and hence capacity for load redistribution whereas the caissons have none.
- The platform pile heads have significant restraint and require two plastic hinges for a mechanism to develop. The caissons are essentially free headed piles and require a single plastic hinge.
- Because the caissons displace more under load, the cyclic loading effects in the soil may be more significant. Further, dynamic amplification and fatigue may also be more important for the caissons.

Because these differences are not well understood caution should be exercised in consideration of these results. It would seem that particular care should be given to the design of nonredundant structures such as caissons

Foundation Axial Capacity. The foundation axial bias factor (B_{fa}) was also determined for the following three possible interpretations of no observable damage to the foundations:

Case D: Pullout/plunging event did not occur in any pile
Case E: Pullout/plunging events did not occur in several piles
Case F: Pullout/plunging event did occur in one pile and others were undamaged

The likelihood functions and posterior distributions were developed for these three cases. The resulting posteriors are given in **Table 3. Fig. 6** shows the likelihood functions for jacket platforms for Case D. The prior and posterior distributions of B_{fa} for Case D are given in **Fig. 7.** The shift in B_{fa} is similar for both 8-leg platforms. The platform P-3 has a minimal effect on the bias factor.

The mean value of the posterior distributions of B_{fa} were similar for Case D and Case E (Case D, mean B_{fa} = 1.73 and Case E, mean B_{fa} = 1.66). The mean value of the posterior distribution of B_{fa} for Case F was higher than the mean value assumed for the prior distribution (i e , 1.3).

These results indicate that both platform foundation mean bias factors are significantly larger than the global mean bias factor of 1.2 determined during Andrew Phase I and that the two foundation bias factors are very dissimilar. The caissons bias factors however are considerably lower. Therefore, to obtain a more realistic representation of platform capacity predictions, the three bias factors should be separated.

Conclusions

1. The predicted foundation mean bias factors were significantly greater than the global mean bias factor determined in Andrew Phase I,[1,3] indicating the importance of separation of bias factors for different failure modes. Based on the selected observations the predictions of platform foundation behavior are more conservative than predictions of overall jacket structure behavior.

2 These bias factors are based on a set (albeit a small sample) of platforms and thus represent a likely trend of biases in the design procedures. They should not be applied to the analysis of specific platforms without further justification. A more thorough estimation of multiple bias factors has been done in the Phase II of Andrew JIP[19] (see companion paper OTC No 8077[20]), where three bias factors (one for the jacket and two for the foundation) were determined from calibration of 9 steel jacket platforms and using a more elaborate reliability analysis and Bayesian updating procedure.

3. The definitions of these bias factors include only foundation survival cases, thus they are likely to be on the higher side. These can be further improved as additional platforms, subjected to extreme loading, are included in calibration. Platforms with foundation damage would be useful as they tend to provide a lower limit to the foundation bias factors. The caisson cases provide a different trend in the foundation lateral bias factor. There is a need for better understanding of the bias factor which can be gained by including suitable jacket platforms.

4. The results of the study suggest that the analysis procedures used for predicting load and resistance are more conservative in the axial mode than in the lateral mode.

5. This study has shown that it is important to assess the platform capacity in different modes and that this can be done by artificially strengthening elements to suppress unwanted failure modes. The possibility of affecting interacting modes should be considered however.

6 There is a need to perform similar studies in other geographical regions to determine the biases in pile foundations in soils, which differ from the Gulf of Mexico platforms

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Appendix—Probability of Failure Formulation

The probability of failure of a platform (given a specified bias B=b) was calculated by numerical integration as follows:

$$P_f(b) = P[\,bR/S < 1\,] = \int_0^{\infty}\{1 - F_s(bx)\} f_R(x)\,dx \quad \text{(A-1)}$$

where f_R is the capacity probability density function (PDF), F_S is the cumulative distribution function (CDF) of load, and b represents a specific value of the bias in the wave force and ultimate capacity procedures. Failure is presumed to be associated with $bR/S < 1$ rather than $R/S < 1$. A lognormal distribution, with a specified median and COV, was assumed for R.

The cumulative density function (CDF) for the maximum base shear, F_S, during the multi-hour (unidirectional) "storm" is obtained as follows

$$F_s(x) = \int_{-\infty}^{\infty}\int_{-\infty}^{\infty} \prod_{hour\ j}^{No.\ of\ Sig.\ Hours} \left\{\int F_{BS}\left(x \mid H = h, U = u_j\right) f_{H|H_S}\left(h \mid H_S = h_{sj}\right) dh\right\}^{N_j} f_{\varepsilon_1}(\varepsilon_1) f_{\varepsilon_2}(\varepsilon_2)\, d\varepsilon_1 d\varepsilon_2 \quad \text{(A-2)}$$

in which N_j denotes the number of random waves in an hour with significant wave height, (h_s) and current (u). ε_1 and ε_2 are the probability density functions (PDFs) of the "errors" in the hindcast of the significant wave heights and currents respectively during Andrew. Note that in this equation $h_{sj} = (H_{sj}' \varepsilon_1)$ and $u_j = (U_j' \varepsilon_2)$, where H_{sj}' and U_j' are the hindcast estimates.

The base shear is calculated from the following model:

$$BS = C_1 [h + C_2 U]^{C_3}\ \varepsilon_0 \quad \text{(A-3)}$$

in which h is a wave height and u is a current, while C_1, C_2 and C_3 are coefficients specific to a particular platform and wave/current direction set (found by fitting this empirical equation to calculated base shears for various pairs of h and u values). ε_0 represents error in base shear estimates, due to wave-to-wave variability, and is assumed to have a log-normal distribution. The probability distribution of each random wave height, H is assumed to follow the empirical Forristall distribution.[22]

Table 1-- Distributions of random variables used in reliability analysis

Item	Nomenclature	Distribution	Expected Value	COV	Remarks
Capacity	R	Log-Normal	per analysis	0.20 0.30	for lateral foundation capacity for axial foundation capacity
Individual wave height	H/H_s	Forristall	per hindcast	per formula	-
Error in significant wave height (H_s)	ε_1	Log-Normal	1 00	0 10	
Error in current (U)	ε_2	Log-Normal	1.00	0 15	
Error in formulation used to compute base shear (S)	ε_s	Log-Normal	1.00	0.20 0.25	case with wave-below-deck (#1) case with wave-in-deck (#1)

(#1) Wave height corresponding to pushover load at platform capacity

Table 2-- Salient characteristics of platforms used in calibration

Platform	Year Installed	Water Depth (ft.)	Number of legs/piles	Pile Diameter (inch)	Performance During Hurricane Andrew
Jacket Platform, P-1	1963	137	8	30	Minor damage to jacket and no damage to foundation
Jacket Platform, P-2	1965	142	8	30	K and X joints teared and no damage to foundation
Jacket Platform, P-3	1969	62	4	36	K-joints teared and no damage to the foundation
Caisson, C-1	1984	35	1	30	Leaned 12 degree
Caisson, C-2	1983	53	1	48	Leaned 15 degree
Caisson, C-3	1983	50	1	48	Leaned 30 degree

Table 3-- Summary of bias factor distributions

Calibration Case	Only Jacket Platforms		Jacket & Caisson Platforms	
	Mean Bias	COV (Error in Bias)	Mean Bias	COV (Error in Bias)
Foundation Lateral Bias Factor, B_g (#1)				
Case A	1.32	0.17	1.09	0 15
Case B	1.26	0 18	1 04	0 16
Case C	1.00	0 16	0.91	0 14
Foundation Axial Bias Factor, B_{ga} (#2)				
Case D	1 73	0 17	-	-
Case E	1.66	0 18	-	-
Case F	1.53	0 18	-	-

(#1) Prior distribution of B_g assumed with mean of 1.0 and COV of 0.30.

(#2) Prior distribution of B_a assumed with mean of 1.3 and COV of 0.30.

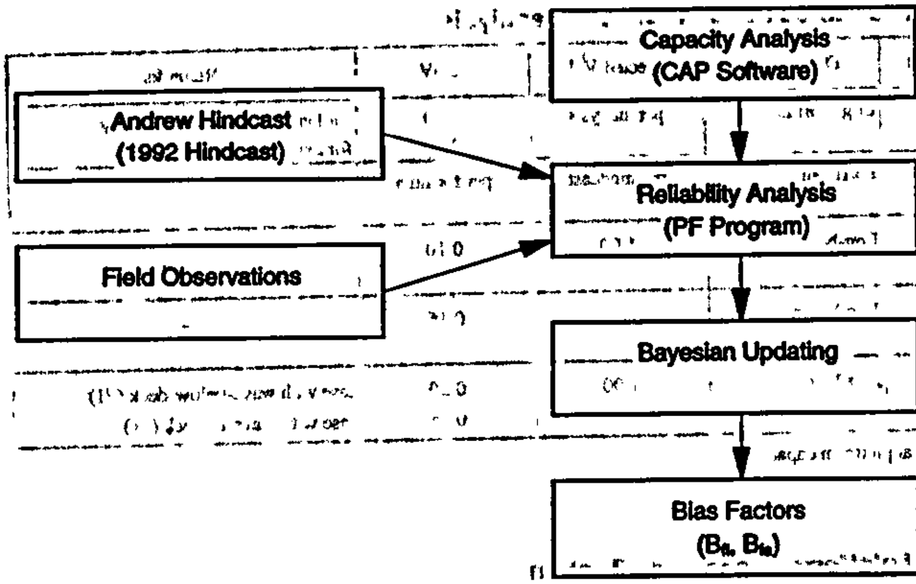

Fig. 1— Calibration Methodology

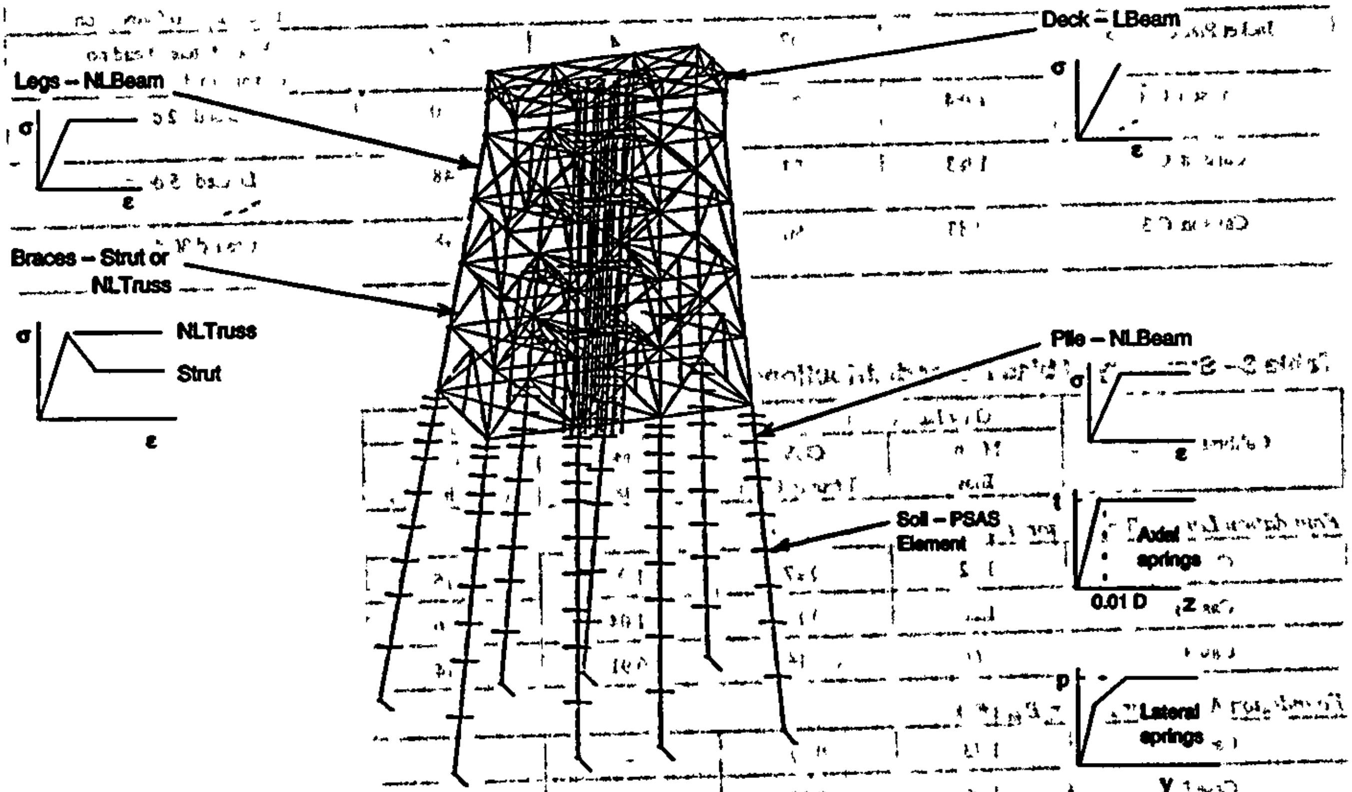

Fig.2— Nonlinear analysis computer model - Platform P-1

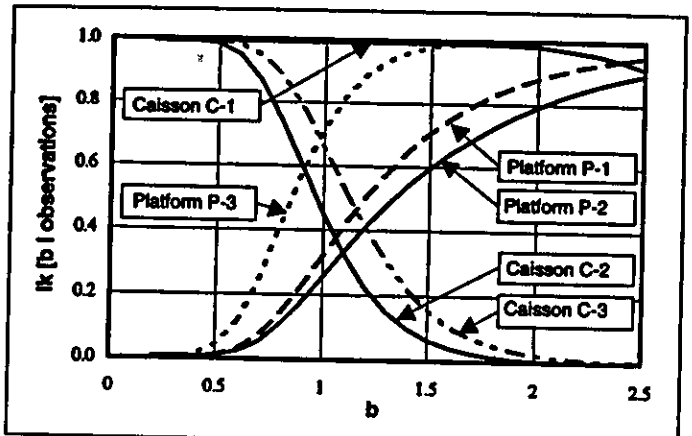

Fig. 3– Likelihood functions for foundation lateral bias, B_s, for calibration Case A

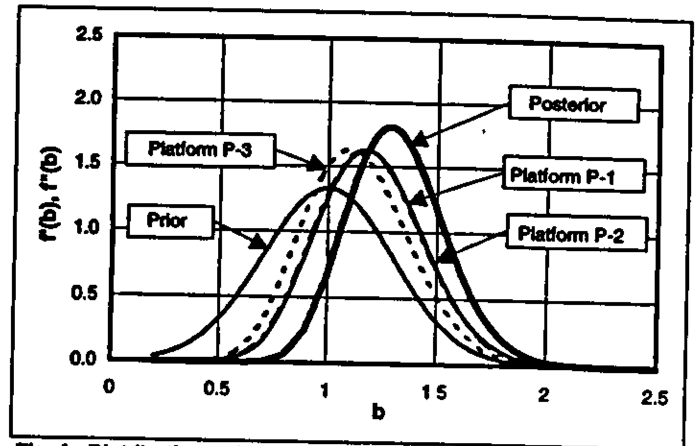

Fig 4– Distributions of bias factor, B_s, due to individual and group of jacket platforms for calibration Case A

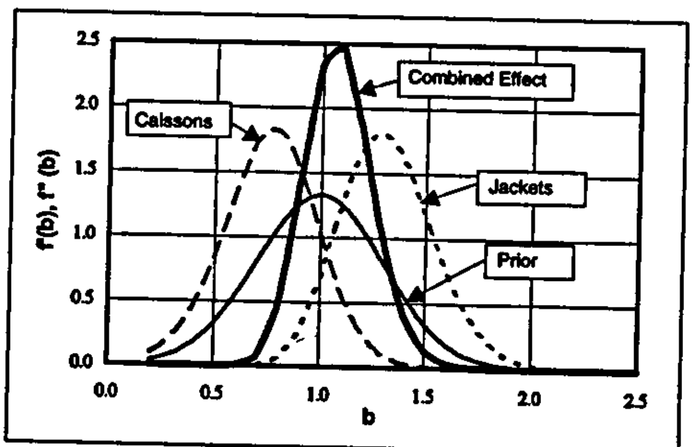

Fig. 5– Distributions of bias factor, B_s, due to all jacket and caisson platforms for calibration Case A

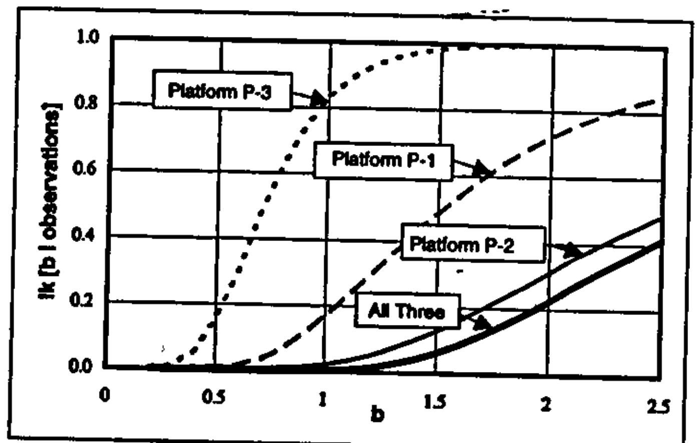

Fig. 6– Likelihood functions for foundation axial bias, B_{fa}, for calibration Case D

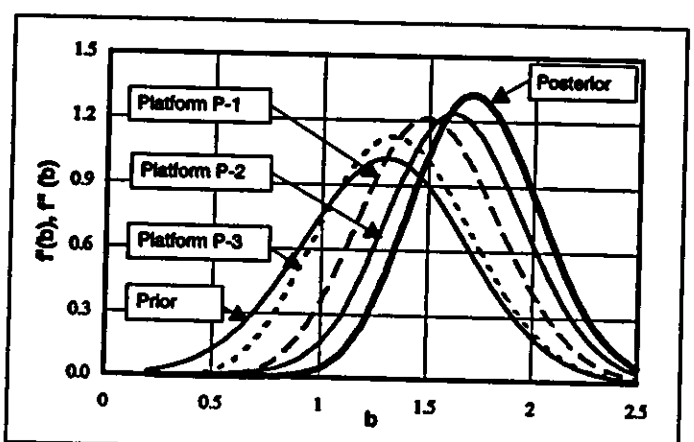

Fig. 7– Distributions of bias factor, B_{fa}, due to individual and group of jacket platforms for calibration Case D

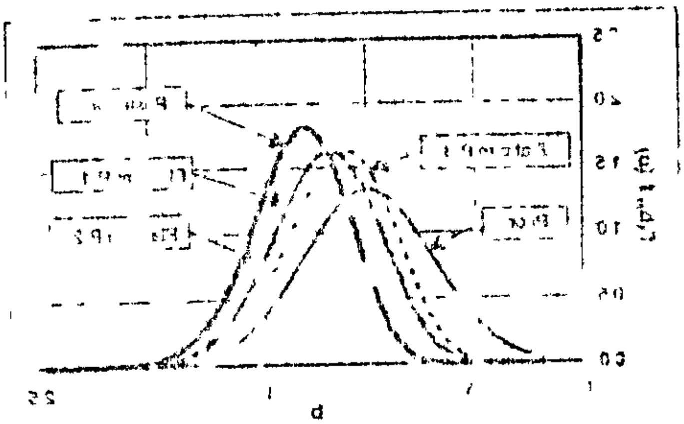

Fig. 4– Distributions of bias factor, B_u ... Case A

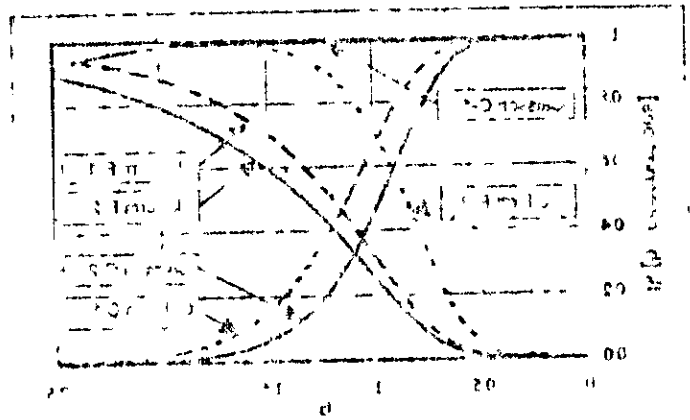

Fig. 3– Likelihood functions for foundation lateral bias, B_u ... Case A

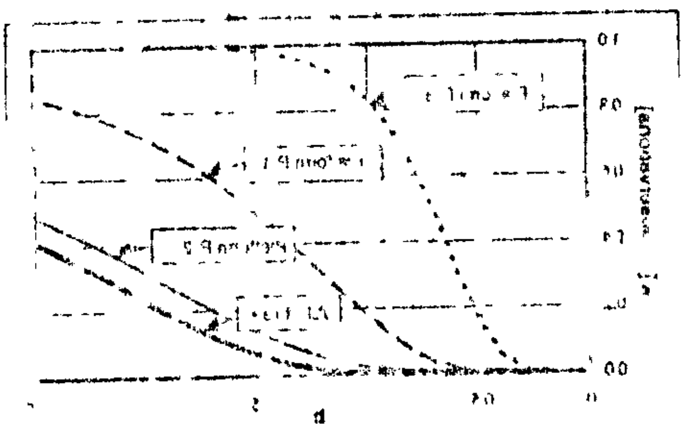

Fig. 5– Likelihood functions for foundation axial bias, B_u, for calibration Case B

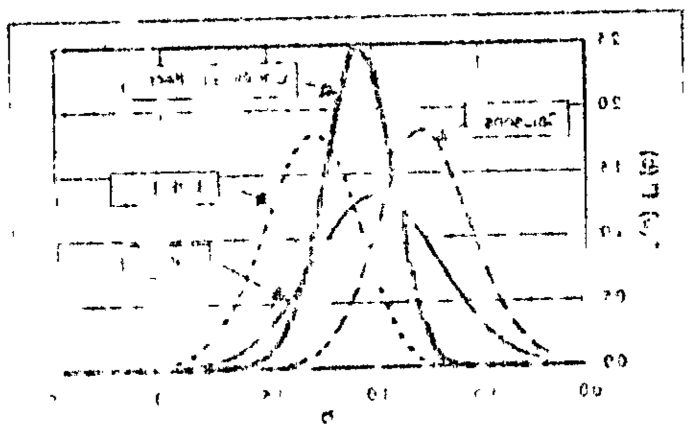

Fig. 6– Distributions of lateral bias, B_u, due to jacket and caisson platforms for calibration Case A

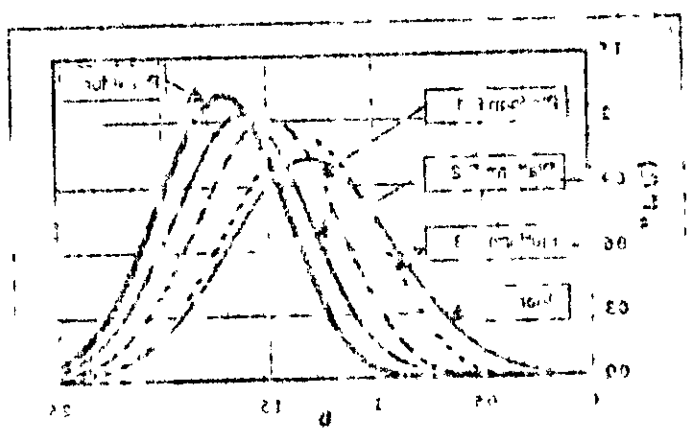

Fig. 7– Distributions of bias factor, B_u ... Case B